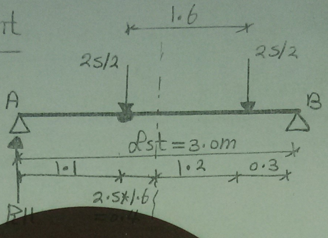
max moment


1.6
25/2
25/2
A
B
$\ell st = 3.0m$
1.1
1.2
0.3
2.5*1.6
= 0.4
RLL

$2.9 < \ell st < 3.5$

$\leq MB = Zero$

$25/2 * 0.3 + 25/2 * 1.9 - RLL * 3 = 0.0 \longrightarrow RLL = 9.16$

$ML.L = 9.167 * 1.1 = 10.1\ t$

impact factor (I)

$I = \phi$ [illegible]

$\phi = 0.73 + \frac{2.1}{\sqrt{LI}}$ [illegible] 0.2

$LI = 3 lst = 3 * 3 = 9.0$
continous

$LI = lst + 3 = 3 + 3 = 6.0$
simple

الرسالة

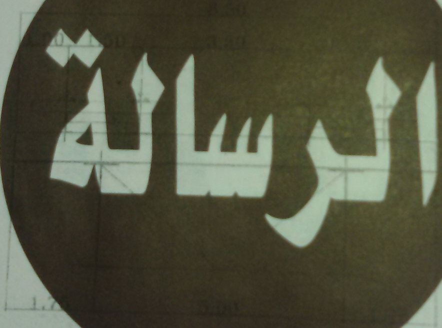
Cairo University
Faculty of Engineering
Civil engineering Dept
Open Book Exam --

Metallic Bridges
Mid. Term Exam 2012
Time: 1.5 Hour
Missing Data May Be Reasonably assumed

A double track open timber floor railway deck bridge, each track carried by two stringers running continuous over cross girders. The main girders are welded plate girders. Total span of bridge is 21.0m divided into seven equal panels (stringer span=3.0m). The double cantilever cross girders are supported on main girder.

1-Find a suitable rolled section for the continuous stringers and design the connection between the stringers and cross girders.
2-Find suitable rolled section for the cantilever cross girder.

الرسالة

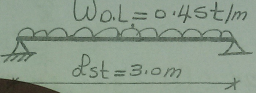


STEEL BRIDGE SECTION

1 : 25

① Design of stringer

***Dead load**


$W_{D.L} = 0.45 t/m$
$\ell_{st} = 3.0 m$

$W_{D.L} = 0.15 + \frac{\text{track weight}}{2}$

$W_{D.L} = 0.15 + \frac{0.6}{2} = 0.45 t/m$

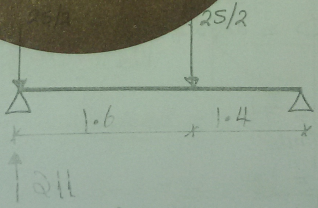
$Q_{D.L} = \frac{0.45 \times 3}{2} = 0.68 t$

$M_{D.L} = \frac{0.45 \times (3)^2}{8} = 0.51 t.m$

الرسالة

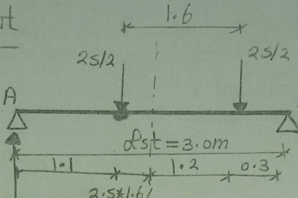
***live load**

max shear


25/2
25/2
1.6
1.4
Q_{LL}

$Q_{LL} = \left[25/2 \times 1.4 + 25/2 \times 3 \right] / 3 = 18.33 t$

max moment


1.6
25/2
25/2
A
B
ℓst = 3.0m
1.1
1.2
0.3

$3.1 < \ell st < 3.5$ RLL $\frac{2.5 * 1.6}{2} = 0.4$

$\sum MB = Zero$

$25/2 * 0.3 + 25/2 * 1.9 - RLL * 3 = 0.0 \longrightarrow RLL = 9.16$

$MLL = 9.167 * 1.1 = 10.1\ t$

الرسالة

impact factor (I)

$I = \phi$ [illegible]

$$\phi = 0.73 + \frac{2.16}{\sqrt{LI} - 0.2}$$

$\underset{continous}{LI} = 3\,Lst = 3 * 3 = 9.0$

$\underset{simple}{LI} = Lst + 3 = 3 + 3 = 6.0$

$$\phi_{cont} = 0.73 + \frac{2.16}{\sqrt{9} - 0.2} = 1.5$$

$$I_{cont} = \phi - 1.0 = 1.5 - 1.0 = 0.5$$

$$\phi_{simple} = 0.73 + \frac{2.16}{\sqrt{6} - 0.2} = 1.69$$

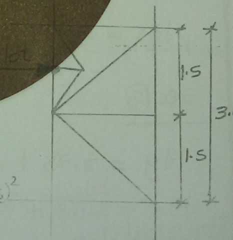
$$I_{simple} = \phi - 1.0 = 1.69 - 1.0 = 0.69$$

$$I_{cont} = 0.5$$
$$I_{simple} = 0.69$$

* lateral shock

force = [illegible]

$Qx = 10t$

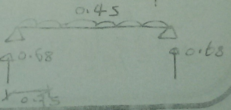


$$My = \frac{10 * 1.5}{4} = 3.75 \ t.m$$

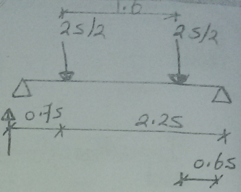
$$M_{DL} = 0.68 * 0.75 - \frac{0.45 * (0.75)^2}{2}$$

at 0.75m from support

$$M_{DL} = 0.38 \ t.m$$



الرسالة


1.6
25/2
25/2
0.75
2.25
0.65

$R = (25/2 * 0.65 + 25/2 * 2.25) / 3 = 12.1$

$M_{LL} = 12.1 * 0.75 = 9.1$ t.m

الرسالة

final straining actions on stringer

a) simple

sec ①

$M_x = 0.51 + 10.1 (1 + 0.69) =$ [illegible] t.m

sec ②

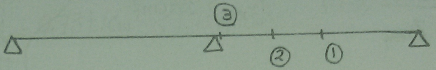
$M_y = 3.75$ t.m

$M'_x = 0.38 + 9.1 (1 + 0.69) = 15.8$ t.m
at support

$Q_y = 0.68 + 18.33 (1 + 0.69) = 31.66$ ton

$Q_x = 10$ t

b) continous stringer



sec ①

$Mx = 0.8 * [0.51 + 10.1 * 1.5] = 12.53$ t.m

$M(LL+I) = 0.8 (10.1 * 1.5) = 12.12$ t.m

الرسالة

sec ②

$My = 3.75$ t.m

$M'x_{(LL+I)} =$ [illegible] $[10.1 * 1.5] = 10.92$ t.m

$Mx' =$ [illegible] $[$ [illegible] $+ 10.1 * 1.5] = 11.22$ t.m

sec ③

$Qx = 10$ t $\qquad Qy = 31.66$ t

$Mx = 0.75 [0.51 + 10.1 * 1.5] = 11.74$ t.m

$M(LL+I) = 0.75 (10.1 * 1.5) = 11.4$ t.m

Design of stringer

Using St.37

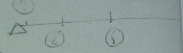
a) Cont. Stringer w Bracing:

$$\frac{13.5\times100}{Z_{x\,req.}} \leqslant 1.4 \qquad Z_{x\,req.} > 964\,cm^3$$

$$\frac{11.2\times100}{Z_x} + \frac{2.25\times100\times6}{Z_x} \leqslant 1.4\times1.2 \qquad Z_x > 1470\,cm^3$$

S.A اللى تم التصميم عليها

هي $Q_x = 6$, $Q_y = 4.90$



$M_x = 13 f_y$

$\bar{M}_x = 11.20$

$M_y = 2.25$

الارقام دي هي هي [illegible]

وهي كلها [illegible]

على محور [illegible]

ومش هتغير التصميم

[illegible]

Try HEB(320)

① Check on Local buckling:

Section from tables

∴ compact w.r.t. local buckling

② Chk on L.T.B:

[illegible] Bracing

Transverse [illegible] bracing

$L_u = 1.5m$

$L_{u_1} = \frac{20\times30}{\sqrt{2.4}} =$ [illegible]

$L_{u_2} = \frac{1380 * 30 * 2.05}{32 * 2.4} =$ [illegible]

∴ compact sec.

③ Chk on stresses:

Sec.(1): $f_x = \frac{135\times100}{1926} = 0.7\,t/cm^2 < 1.54\,t/cm^2$ O.K

الرسالة

Sec.(2):

$$f_x = \frac{11.2 \times 100}{1926} = 0.58\,t/cm^2 < 1.54\,t/cm^2$$

$$f_y = \frac{2.25 \times 100}{616/2} = 0.73\,t/cm^2 < 1.728\,t/cm^2$$

$$\frac{f_x}{F_x} + \frac{f_y}{F_y} < 1.2 \qquad \frac{0.58}{1.54} + \frac{0.73}{1.728} = 0.8 < 1.2 \therefore O.K.$$

وجدي؟ اصلا لم نعمل [illegible]

ربنا ستر معه

$$f_{act} = \frac{M_x}{Z_x} + \frac{M_y}{Z_y/2} < f_{ball} \times 1.20$$

Case I

compact or non compact.

الرسالة

Sec.(3):

$$\frac{12.7 \times 100}{0.85 \times 1926} = 0.78\,t/cm^2 < [illegible]\,t/cm^2 \quad O.K.$$

$$\frac{29.9}{0.85 \times 32 \times 1.15} = 0.96\,t/cm^2 > 0.84\,t/cm^2 \quad \therefore \text{unsafe}$$

HEB(36): $q_y = \frac{29.9}{0.85 \times 36 \times 1.25} = 0.78\,t/cm^2 < 0.84\,t/cm^2$

O.K.

$$q_x = 1.5 \times \frac{6}{30 \times 2.25} = 0.13\,t/cm^2 < 1.2 \times 0.84$$

O.K.

$$f_x = \frac{12.7*100}{0.85*2399} = 0.62\,t/cm^2$$

Value at web top: $f_x = \frac{12.7*100*\left(\frac{36}{2}-2*2.25\right)}{0.85*43190} = 0.47\,t/cm^2$

Equivalent Stress: $f_{eq} = \sqrt{(0.47)^2+3(0.78)^2} = 1.43\ t/cm^2 < 1.1*1.54$

O.K.

☆ ١١ هنا يا كم

$\leftarrow f_{act}$

Check on Deflection:

$$M_{L.L.} = \frac{13.1}{1.75} = 7.5\,t.m = \frac{W_{eq}(3)^2}{8} \longrightarrow W_{eq.} = 6.67\,t/m'$$

$$\Delta_{act.} = \frac{5}{384}\frac{6.67*(300)^4}{100*2100*43190} = 0.08\,cm$$

$$\Delta_{all.} = \frac{300}{800} = 0.38\,cm > \Delta_{act.}$$

∴ safe

∴ Used section: HEB(360)

الرسالة

Design of connection bt stringers & X.G.

Use M20 (10.9) - Bearing type bolts

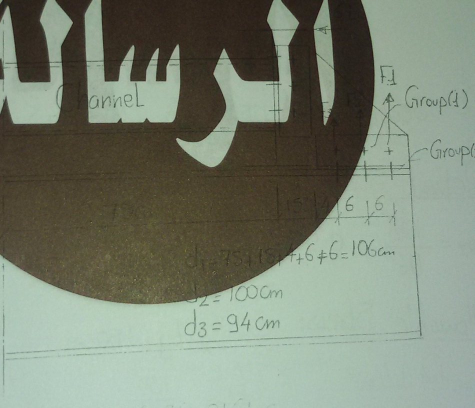
* Stringer section: HEB(36)

$R_{d.sh.} = 2 * 0.2 * 10 * \frac{\pi(2)^2}{4} = 12.6t$

$R_{bearing} = 0.8 * 2 * 3.6 * 1 = 5.8t$ [illegible] d

$R_{s.sh.} =$ [illegible]

الرسالة


Channel
F1
F2
Group(1)
Group(
15
4
6
6

$d_1 = 75 + 15 + 4 + 6 + 6 = 106 cm$

$d_2 = 100 cm$

$d_3 = 94 cm$

Group(1): $M = 6 \times h_{stringer} = 6 * 36 = 216 t.cm$

$$F_1 = \frac{M.d_1}{\Sigma d^2} = \frac{216 \times 106}{2[94^2 + 100^2 + 106^2]} = 0.38 ton$$

Approx Sec:

$\frac{M}{Z} \leq 1 t/cm^2$ $\frac{108}{Z_x} \leq 1$ $Z_x > 108 cm^3$

Try UPN(160)

Chk on Local Buckling:

- section from table $\rightsquigarrow$ ∴ compact w.r.t. Local buc

Chk on L.T.B:

$L_u = 150 cm$ $L_{u_1} = \frac{20 \times 6.5}{\sqrt{2.4}} = 84 cm$

$L_{u_2} = \frac{1380 \times 6.5 \times 1.05}{16 \times 2.4} = 245 cm$

Section is unsymm. & $L_{u_1} < L_u < L_{u_2} \Rightarrow$ Non-com

الرسالة

Actual Stress:

$f_{bx} = \frac{108}{116} = 0.93 t/cm^2$ & $f_a = \frac{3}{24} = 0.13 t/cm^2$

Allowable Stress:

$L_x = L_y = 150 cm$

$F_{bx} = 1.4 t/cm^2$ & $\frac{L_x}{i_x} = \frac{150}{6.21}$ $\frac{L_y}{i_y} = \frac{150}{1.89} = 79.4$

$F_a = 1.4 - 0.000065(79.4)^2 = 0.99 t/cm^2$

$f_a/F_a = \frac{0.13}{0.99} = 0.13 < 0.15 \rightsquigarrow A_1 = 1$

Chk on Stresses:

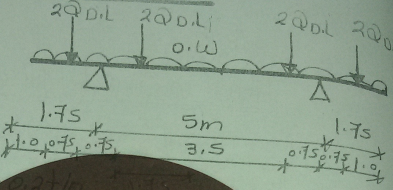
$$\frac{0.93}{1.4} + \frac{0.13}{0.99} = 0.8 < 1.0 \qquad \text{O.k.}$$

Used section: ⊔PN(160)

الرسالة

② Design of (X.G)

(D.L)

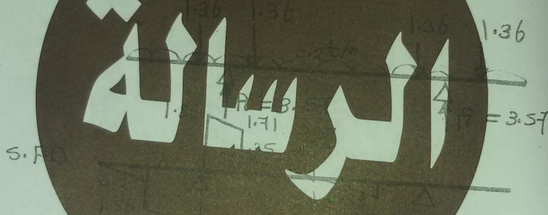

2Q D.L
2Q D.L
O.W
2Q D.L
2Q D.L
1.75
5m
1.75
1.0
0.75
0.75
3.5
0.75
0.75
1.0
1.75

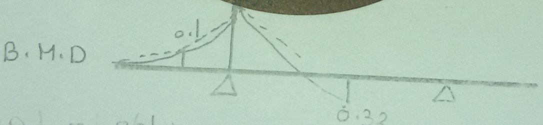
assume O.W = 0.2 t/m

2DL = 0.68

2Q D.L = 1.36t

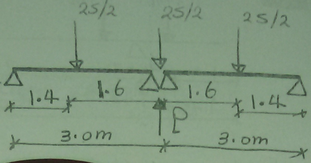
الرسالة


1.36
1.36
1.36
1.36
0.2 t/m
R = 3.57
R = 3.57
1.71
S.F.D



2Q D.L = 1.86t
MD.L = 1.33 t.m) at support

(L.L)


25/2
25/2
25/2
1.4
1.6
1.6
1.4
P
3.0m
3.0m

$P = 25/2 + 2 * \frac{25}{2} * 1.4/3 = 24.167$ ton

$I = \phi - 1.0$

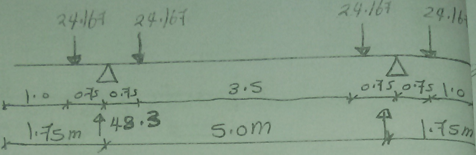
$\phi = 0.73 + \frac{2.16}{\sqrt{L_I} - 0.2}$

$L_I = 2L_x = 2 * 5 = 10.0m$

$\phi = 0.73 + \frac{2.16}{\sqrt{10} - 0.2} = 1.46$

الرسالة

$I = \phi - 1.0 = 1.46 - 1.0 = 0.46$


24.167
24.167
24.167
24.167
1.0
0.75
0.75
3.5
0.75
0.75
1.0
1.75m
48.3
5.0m
1.75m

$Q_{L.L} = 24.167\,t$

$M_{L.L\ \text{at support}} = 24.167 \times 0.75 = 18.13\ t.m$

assume B.F. Bracing is used $M_y = 0.0$

⁰⁰ final straining actions

$M_x = 1.33 + 18.13\,(1+0$ [illegible] $27.8\ t.m$

$Q_y = 1.86 + 24.167\,(1+0.46) = 37.14\ t$

a) approximate sec

$29.$[illegible]$/Z_x \leq 1.4 \qquad Z_x \geq 2118\ cm^3$

try HEB (340)

b) check local Buckling → sec from table
no check of local Buckling

الرسالة

$L_u = 2.1 * 1.75 = 3.7\,m$ $\qquad L_{u1} = \frac{20 * 30}{\sqrt{2.4}} = 387\,cm$

$L_u < L_{u1}$ $\qquad$ ∴ compact sec. $\qquad L_{u2} = \frac{1380 * 30}{34 * 2.4}$

$< L_{u2}$ $\qquad = 1090.84$

⊙ Chk on Stresses:

$f_x = \frac{29.7 * 100}{0.85 * 2156} = 1.62\, t/cm^2 > 1.54\, t/cm^2$ O.k.

Use HEB(360) $f_x = \frac{29.7 \times 100}{0.85 \times 2399} = 1.46\, t/cm^2 < 1.54\, t/cm^2$

O.K.

الرسالة

$q_y = \frac{36}{0.85 \times 36 \times 1.25} = 0.94\, t/cm^2 > 0.84$

Try HEB(4

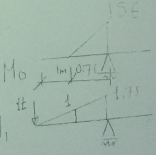
$q_y = \frac{36}{0.85 * 40 \times 1.35} = 0.78\, t/cm^2 < 0.84\, t/cm^2$

safe

⊙ Chk on deflection:

- Deflection at tip of cantilever due to L.L.

$\Delta_{act} = \frac{1}{EI} \int M_1 M_0 \, dl = \frac{1}{EI} \left[\frac{0.75}{6} \left(2 \times 15.6 \times 1.75 + 15.6 \right) \right]$



⑱

$$\Delta_{act.} = \frac{13.9}{EI} = \frac{8.8 \times 100^3}{2100 \times 57680} = 0.072\ cm$$

- Deflection under the concentrated load

$$\Delta_{act.} = \frac{1}{EI}\left[\frac{0.75}{6}\left(2 \times 15.6\right)\right] = 0.03\ cm$$

$$\Delta_{all.} = \frac{175}{180} = 0.97\ cm > \Delta_{act.} \quad \therefore O.K.$$

Used section : HEB (400)

الرسالة

Cairo University
Faculty of Engineering
Civil engineering Dept.
Open Book Exam. --

Metallic Bridges
Mid. Term Exam.2011
Time: 1.5 Hour
Missing Data May Be Reasonably assumed

Figure (1) shown the cross section of 40m simply supported two ways through roadway plate Girder Bridge. The bridge floor consists of a 22 Cm reinforced concrete slab carried on top of cross girders spaced at 4 m, the cross girders are simply supported on main girder with span 7.75m.

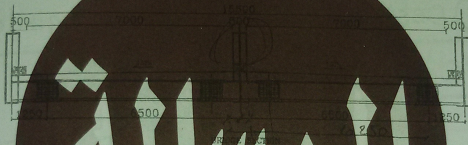


FIG. (1)

الرسالة

1-Find a suitable rolled steel section for the cross girder.
2-Find suitable built-up section for intermediate main girder due to the following data.

M (dead load) = 400mt
M (live load) = 1500mt
Q (dead load) = 80t
Q (live load) = 120t

⑳

4.25

Midterm(2011)

simply supported, Roadway, through Brid

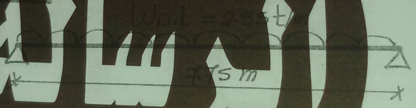
$t_{R.C} = 22\ cm$

$S = 4.0\ m$

X.G is simply supported $L_{X.G} = 7.75\ m$

① Design of rolled sec for Cross girder

I Dead load



الرسالة

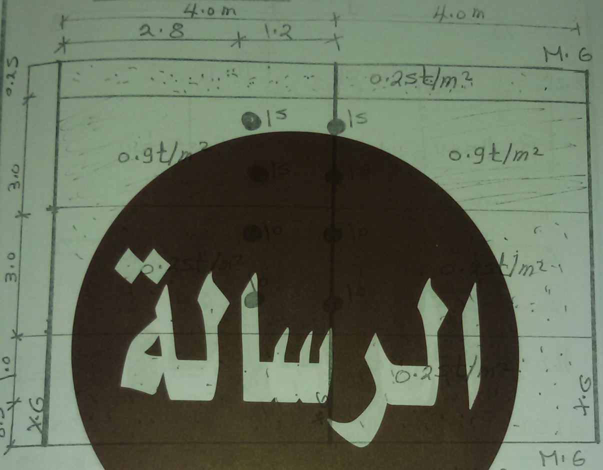
$W_{D.L} = o.w_{X.G} + [\gamma_{R.C} * t_{R.C} + \gamma$ [illegible] $* t_{0.s}] * S$

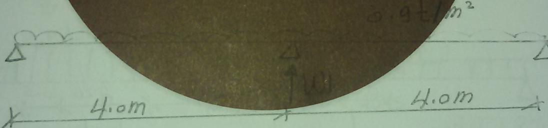
$W_{D.L} = 0.25 + [2.5 * 0.22 +$ [illegible] $* 0.05] * 4 = 2.85$

$Q_{y\,D.L} = 2.85 * 7.75 / 2 = 11.04\ ton$

$M_{x\,D.L} = 2.85 * (7.75)^2 / 8 = 21.4\ t.m$

2 live load

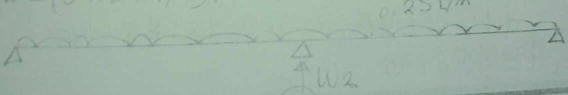

4.0m
2.8
1.2
4.0m
M.G
0.25
0.25t/m²
15
15
0.9t/m²
0.9t/m²
3.0
15
15
10
10
3.0
0.25t/m²
0.25t/m²
10
10
1.0
0.25t/m²
0.5
X.G
X.G
M.G

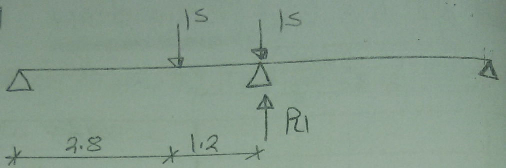

0.9t/m²
W1
4.0m
4.0m

الرسالة

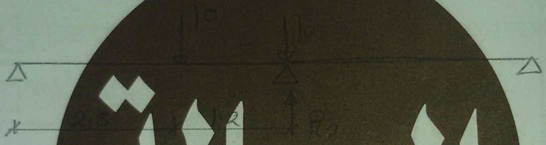
$W_1 = (0.9 * 4/2) * 2 = 3.6$ ton/m

$W_2 = (0.25 * 4/2) * 2 = 1.0$ t/m



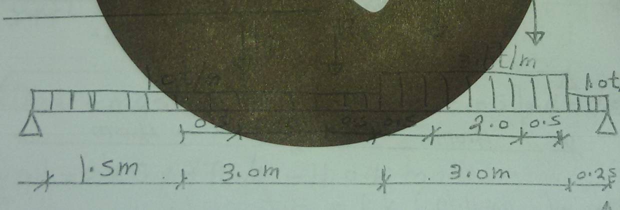

15
15
R1
2.8
1.2

$R_1 = (15*2.8 + 15*4)/4 = 25.5$ ton


10
10
2.8
1.2
R2

$R_2 = (10*2.8 + 10*4)/4 = 17$ t..n

.a) case of max shear


25.5
t/m
1.0t/m
1.0t
0.5
0.5
0.5
2.0
0.5
1.5m
3.0m
3.0m
0.25

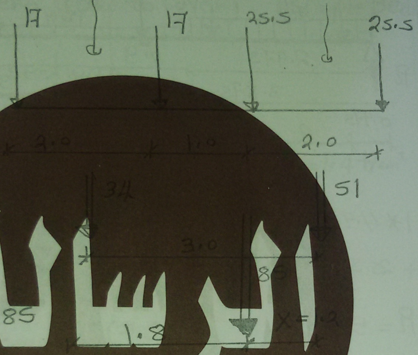
$R_{1L} = [1.0*1.5*2.25 + 3.6*3*6 + 1.0*0.25*7.125$

$+ 17*2 + 17*4 + 25.5*5 + 25.5*7]/7.25$

$R_{1L} = 62.6$ ton

الرسالة

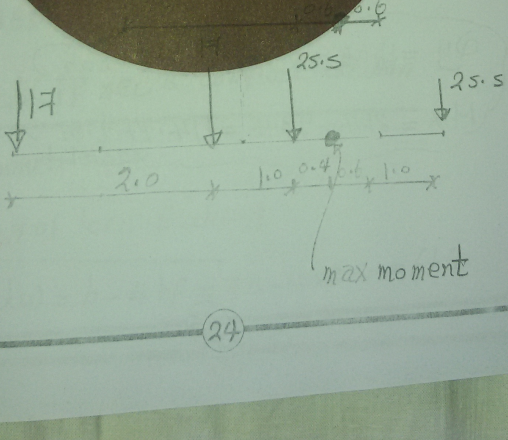
b) case of max moment

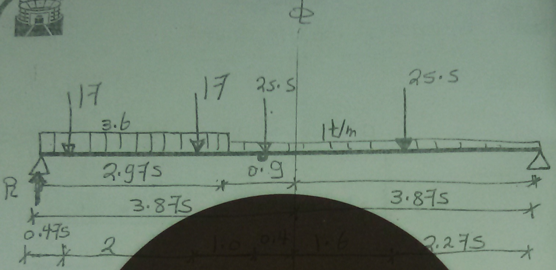

17
17
25.5
25.5
2.0
1.0
2.0
34
51
3.0
85
1.8
X = 1.2

R = 51 + 34 = 85

85 * X - 34 * 3 = 0.0 → X = 1.2

الرسالة


0.6
0.6
17
25.5
17
25.5
2.0
1.0
0.4
0.6
1.0
max moment


℄
17
17
25.5
25.5
3.6
1t/m
R
2.975
0.9
3.875
3.875
0.475
2
1.0
0.4
1.6
2.275

1*4.775*2.3875+3.6*2.975*1.4875+25.5*2.2

+25.5*4.275+17*5.275+17*7.275-7.75 R=0.0

R=52.61

MLL = 52.61*3.475-17*1.0 [illegible] -1*0.5 *0.25 - 3.6 *2.9 [illegible] = [illegible] t.m

Qy =11.04 + [illegible] = [illegible] ton

Mx = 21.4+93.4 =114.14 t.m

الرسالة

3 Design of section

* Use steel (37)

* L_u = zero (due to cover) (simple Beam)

ⓐ choice of section

$f = \frac{M_x}{Z_x} \ngtr f_{all} = 1.4\ t/cm^2$

assume Non compact

$f = \frac{128.4 \times 100}{Z_x} \ngtr 1.4$

[illegible]

$Z_x \nless 9152.85\ cm^2$

try B.F.I No. (750)

$h = 75$ $b = 30$ $t_w = 1.8$ $t_f = 3.4$

$I_x = 31630$[illegible] cm^4 $Z_x = 8425\ cm^3$

الرسالة

ⓑ check if sec is compact or Non

* symmetric sec

* check of local Buckling

web $d_w = h - 4t_f = 75 - 4 \times 3.4 = 61.4$

$d_w/t_w = 61.4/1.8 = 34.11 < 82$

Flange

$C = (b_f - t_w - 2t_f)/2$

$C = (30 - 1.8 - 2*3.4)/2 = 10.7$

$C/t_f = 10.7/3.4 = 3.14 < 10.9$

∴ local Buckling is safe

* unsupported length

$L_u = \text{zero} < L_{u1}$
$< L_{u2}$

∴ lateral torsional buckling is safe
and section is compacted sec.

Ⓒ check of Bending stresses

$f = \frac{M_x}{Z_x} = \frac{128.14*100}{8435} = 1.52\ t/cm^2 < 1.54$

safe

الرسالة

ⓓ check of shear

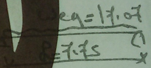
$$q_y = \frac{Q_y}{A_{web}} \xrightarrow{73.64} 0.84$$

$$q_y = \frac{67.24}{75 * 1.8} = 0.49 < 0.84\ t/cm^2$$

(safe)

ⓔ check of Deflection

$$M_x = 128.14 = \frac{W_{eq} * (7.75)^2}{8}$$



$$\text{get } W_{eq} = 17.07\ t/m$$

$$\delta_{act} = \frac{5 W_{eq} * \ell^4}{384 E I_x}$$

$$\delta_{act} = \frac{5 * 17.07/100 * (775)^4}{384 * 2100 * 316300} = 1.2\ cm$$

$$\delta_{all} = \frac{span}{800} = \frac{7.75 * 100}{800} = 0.97\ cm$$

$\delta_{act} > \delta_{all} \longrightarrow$ try sec No (850)

$$\delta_{act} = \frac{1.2 * 316300}{443900} = 0.86 < 0.97$$

(safe)

الرسالة

ⓕ check of lateral torsional Buckling duri
construction

$l_u = L_{x.G} = 7.75m$ (No co

$M = M_{D.L} = 21.4\ t.m$

$f_c = \frac{M_{D.L}}{Z_x} = \frac{31.4*100}{10445} = 0.204\ t/cm^2$

B.F.I (850)

$F.L.T.B ① = \frac{800 A_f c_b}{l_u * d} = \frac{800*30*3.6*1.0}{775*(85-2*3.6)} =$

$\therefore f_c < F.L.T.B ①$ ∴ compression is sa
in construction
[illegible]ge

الرسالة

ⓕ check of lateral torsional Buckling duri

construction

$lu = \ell_{x.G} = 7.75m$ (No co

$M = M_{D.L} = 21.4\ t.m$

$f_c = \frac{M_{D.L}}{Z_X} = \frac{31.4*100}{10445} = 0.304\ t/cm^2$

S.F.I (850)

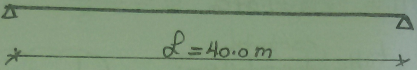
$F.L.T.B_{①} = \frac{800 A_f c_b}{lu*d} = \frac{800*30*3.6*1.0}{775*(85-2*3.6)} = 1.$[illegible]

$\therefore f_c < F.L.T.B_{①}$ ∴ compression is safe in construction stage

الرسالة

2 main girder

steel (52)



$M_{D.L} = 400\ t.m$

$M_{L.L} = 1500\ t.m$

$Q_{D.L} = 80\ ton$

$Q_{LL} = 120\ ton$

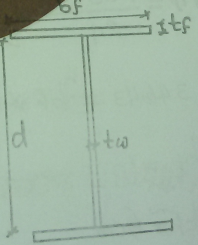
$I = 0.4 - 0.008 \times 40 = 0.08 \longrightarrow 0.15$

$Q_y = Q_{D.L} + Q_{LL}(1+I) = 80 + 120(1+0.15) = 218\ ton$

$M_x = M_{DL} + M_{LL}(1+I) = 400 + 1500(1+0.15) = 2125\ t.m$

الرسالة

ⓐ approximate sec



$d = 0.25\sqrt[3]{\frac{M_x \to 2125}{f_t \to 2.1}}$

$d = 2.5\ m$

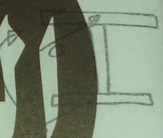
$d \not< span/20 = 40/20 = 2.0\ m$

$$(t_w)^2 = \frac{q_{max}}{41.65\sqrt{F_y}} = \frac{218}{41.65\sqrt{3.6}} = 2.$$

$t_w = 1.67\,cm$

take $\boxed{t_w = 1.4\,cm}$ $\boxed{d = 250\,cm}$

to get flange section



$$T = C = \frac{M_{max}}{d} = \frac{2125}{2.5}$$

$T = C = 850$ ton

$$A_f = \frac{C}{F_t} - \frac{1}{6}\,d\,t_w$$

$$A_f = \frac{850}{2.1} - \frac{1}{6} * 250 * 1.4 = 346.43\,cm^2$$

$346.43 = b_f * t_f$

$b_f = (0.2 \rightarrow 0.3)\,d = 50 \rightarrow 75 \rightarrow$ $b_f = 6$

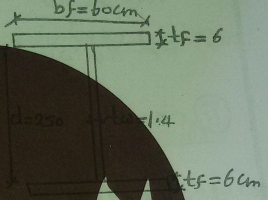
$t_f \cong 6\,cm$

الرسالة

check of local Buckling

$$\frac{bf/2}{tf} = \frac{60/2}{6} = 5 < 21/\sqrt{3.6} = 11$$

safe



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ⓑ check of shear (Buckling due to shear)

ⓘ assume Unstiffned

$$d/tw = 250/1.4 = 178.6 > 159/\sqrt{3.6} = 83.8$$

$$\therefore q_b = \left[119/\left(\frac{d}{tw}\right)\sqrt{Fy}\right] * 0.35\,fy$$

$$q_b = \frac{119}{178.6 * \sqrt{3.6}} * 0.35 * 3.6 = 0.44\, t/cm^2$$

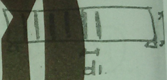
$$q_{act} = \frac{Q_y}{d * t_w} = \frac{218}{250 * 1.4} = 0.62 t/c$$

$q_{act} > q_b$ ∴ the web is unsafe

use V.l stiffner at every distance $d_1 =$

(ii) check of case of use V.l stiffners

$$\alpha = \frac{d_1}{d} = \frac{2.0}{2.5} = 0.8 < 1.0$$



$$k_q = 4 + 5.34/\alpha^2 = 12.34$$

$$\lambda_q = \frac{d}{t_w}\sqrt{\frac{F_y/\sqrt{3}}{1898 \cdot k_q}}$$

$$\lambda_q = \frac{250}{1.4}\sqrt{\frac{3.6/\sqrt{3}}{1898 * 12.34}} = 1.68 > 1.2$$

$$q_b = \frac{0.9}{\lambda_q} * 0.35 F_y = \frac{0.9}{1.68} * 0.35 * 3.6 = 0.6 t/$$

$q_{act} = 0.62 < q_b$

الرسالة

© check of Buckling due to Bending

$d/t_w = 250/1.4 = 178.6 \quad > \quad 190/\sqrt{3.6} = 100$

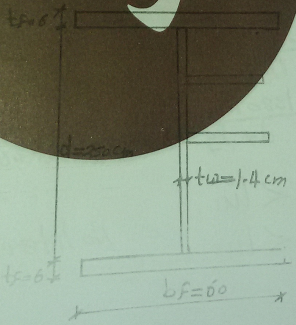
∴ Use H.S stiffner at (0.2d) from compression flange and check

$d/t_w = 178.6 \quad > \quad 320/\sqrt{3.6} = 168.7$

∴ Use another H.S stiffner at (0.5d) from comp. flange

$d/t_w = 178.6 \quad < \quad 365/\sqrt{3.6} = 192.4$ $\underline{\underline{\text{ok}}}$

الرسالة



ⓓ check of Bending stresses

$$I_x = \frac{1.4 * (250)^3}{12} + 2 * 60 * 6 * \left(\frac{250}{2} + \frac{6}{2}\right)^2 = 13619396.$$

$$f_t = \frac{M_x}{I_x} y_{max} = \frac{2125 * 100}{13619396.67} * (125 + 6) = 2.04 < 2.1$$

S

ⓔ check of lateral torsional Buckling

الرسالة

$$L_u = 1.25 * \underset{4.0}{S} = 5m$$

$$L_u = 500 cm$$

$$L_{u1} = \frac{20 b_f}{\sqrt{F_y}} = \frac{20 * 60}{\sqrt{3.6}} = [illegible]$$

$$L_{u2} = \frac{1380 A_f C_b}{d * F_y} = \frac{[illegible] * 1.0}{[illegible] * 3.6} = 552 cm$$

$L_u < L_{u1}$

$< L_{u2}$ ∴ No lateral torsional Buck

Ⓕ check of Fatigue

$$f_{Fatigue} = \frac{M_{LL+I}}{I_x} * y_{max} * \underset{\text{Roadway}}{\underset{\downarrow}{0.5}} =$$

$$f_{Fatigue} = \frac{1725 * 100}{1361939\ldots} * (125+6) * 0.5 = 0.829\ t/cm^2$$

* classe (B') ⟶ [illegible] 1.02 t/cm²

* No of cycles = 2×10^6

$f_{Fatigue} = 0.829 <$ [illegible] (safe)

الرسالة

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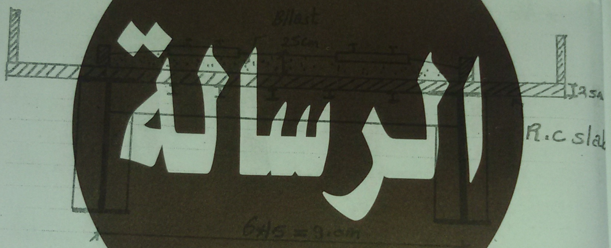
Midterm

ذى المسئلة اللى فاتت
بالضبط (مهم جداً)

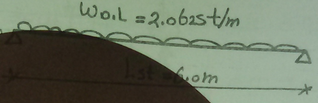


a double track Railway Deck Bllasted floor Bridge with (25 R.C thick) and (25cm Bllast thick) spacing between X.G is equal to 6.0m, the stringer is continously supported on (X.G)

(A) Design Rolled section of an intermediate stringer together with its connection to the cross girder (steel 52)

I straining actions on stringer

ⓐ Dead load



$W_{OL} = o.(\omega) + \frac{\text{track wight}}{2} + \left[\gamma_c * t_s + \gamma_{ballast} * t_{Blast}\right] * d_o$

$W_{OL} = 0.15 + \frac{0.6}{2} + \left[2.5*0.25 + 1.8*0.25\right] * 1.5 = 2.0625 t/$

$Q_{DL} = W_{DL} * L_{st}/2 = 2.0625*6/2 = 6.1875$ ton

$M_{D.L} = W_{DL} * (L_{st})^2/8 = 2.0625*(6)^2/8 = 9.3$ t.m

ⓑ live load

impact factor (I)

$$I = \phi - 1.0$$

$$\phi = 0.73 + \frac{2.16}{\sqrt{L_I} - 0.2}$$

$L_I = 2L_{st} = 2*6 = 12$

$\phi = 1.39$

$I = 1.39 - 1.0 = \boxed{0.39}$

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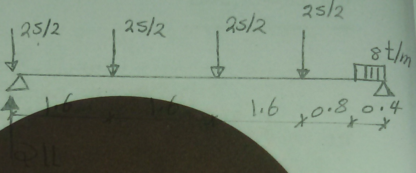
Case of max shear

$l_{st} = 6.0m$


25/2
25/2
25/2
25/2
8 t/m
1.6
1.6
1.6
0.8
0.4
R11

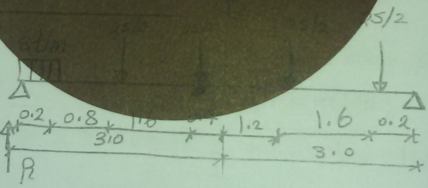
$8*0.4*0.2+(25/2)*1.2+(25/2)*2.8+(25/2)*4$

$+(25/2)*6-6\,R_{11}=0.0$

$R_{11}=30.11$ ton

الرسالة

Case of max moment


8t/m
25/2
25/2
25/2
25/2
0.2
0.8
1.6
1.2
1.6
0.2
3.0
3.0
R

$R=\left[(25/2)*0.2+(25/2)*1.8+(25/2)*3.4+(25/2)*\right.$

$\left.+8*0.2*5.9\right]/6 = 23.24$ ton

$M_{LL} = 23.24 * 2.6 - (25/2) * 1.6 - 8 * 0.2 * 2.5$

$M_{LL} = 36.42 \ t.m$

$Q_y = 6.1875 + 30.11 (1 + 0.39) = 48.04 \ ton$

$M_X = 9.3 + 36.42 (1 + 0.39) = 59.92 \ t.m$

* due to R.c slab → No effect of lateral load → $Q_X = 0.0$

$M_y = 0.0$

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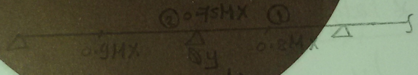
in case of continous stringer



2 Design of stringer section

ⓐ approximate

sec ①

→ assume Non compact

$$f = \frac{0.8 M_X}{Z_X} \not> 2.1$$

$$f = \frac{0.8 * 59.92 * 100}{Z_X} \not> 2.1 \rightarrow Z_X \not< 2282.67$$

try HEA No(450)

ⓑ check If compacted oR Non compact

* symmetric sec

* Check of local Buckling

web

$dw = h - 4tf = 44 - 4*2.1 = 35.6$

$dw/tw = 35.6/1.15 = 30.96 < 127/\sqrt{3.6} = 66.9$

flange

$c = (bf - tw - 2tf)/2 = (30 - 1.15 - 2*2.1)/2$

$C = 12.325$

$c/tf = 12.325/2.1 = 5.87 < 16.9/\sqrt{3.6} = 8.9$

الرسالة

* lateral torsional Buckling

$Lu = 0.2 Lst = 0.2*600 = 120\,cm$

$$Lu_1 = \frac{20\,bf}{\sqrt{fy}} = \frac{20*30}{\sqrt{3.6}} = 316.23\,cm$$

$$Lu_2 = \frac{1380\,\overset{30*2.1}{Af}*\overset{}{cb}^{=1.0}}{\underset{(44-2*2.1)}{d}*\underset{3.6}{fy}} = 606.78\,cm$$

$Lu < Lu_1$ → sec is compacted

$< Lu_2$

Ⓒ check of Bending stresses sec ①

$$f = \frac{0.8\,Mx}{Zx} = \frac{0.8 * 63.99 * 100}{2896} = 1.77 < 0.64\,fy$$

2.3

safe

ⓓ check of stresses of sec ②

normal

$$f = \frac{0.75\,Mx}{0.85 * Zx} = \frac{0.75 * 63.99 * 100}{0.85 * 2896} = 1.95 < 2.3$$

↳ Bolts

shear

$$q_y = \frac{Q_y}{A_{web} * 0.85} = \frac{50.5}{0.85 * 49 * 1.15} = 1.17 < 1.26$$

equivlent

$$f_{eq} = \sqrt{(f)^2 + 3(q)^2} \ngtr f_{all} * 1.1$$

$$f_{eq} = \sqrt{(1.95)^2 + 3(1.17)^2} = 2.81 > \begin{matrix} 2.3 * 1.1 \\ 2.53 \end{matrix}$$

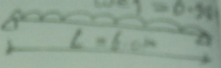
(Un safe)

try HEA No(550)

الرسالة

© Check of Deflection

$M_{LL} = 31.25\ t.m$



$31.25 = \frac{W_{eq} * (6)^2}{8} \longrightarrow w_{eq} = 6.94$

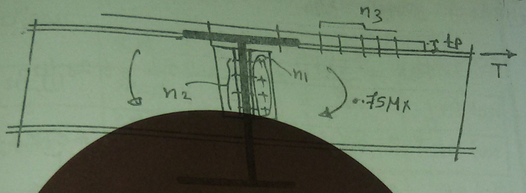
$\delta_{ad} = \frac{5\, W_{eq} * l^4}{384\, E\, I_x}$ [illegible] $= \frac{5 * (6.94/[illegible]) * (600)^4}{384 * 2[illegible] * 1[illegible]900}$

$\delta_{ad} = 0.49$ [illegible] $<$ [illegible] $= 0.$[illegible]

Safe

الرسالة

Design of connection between stringer and (X.G)



group ⓝ1

. Use H.S.B (10.9) M20 Bearing type

. double shear m=2.0

$R_{sh} = \frac{\pi (2)^2}{4} * 2 * 0.2 * 10 = 12.56$ ton

$R_b = 2 * 1.25 * \overset{tw}{0.8} * 5.2 = 10.4$ ton ✓

$n_1 = \frac{50.5}{10.4} \cong$ 5 Bolts

group ⓝ2

. single shear

$R_{sh} = 6.28$ ton $< R_b$

$n_2 = \frac{50.5}{6.28} \cong$ 10 Bolts

$n_2/2 = 5.0$ Bolts

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group (n_3)

single shear

$R_{sh} = \frac{\pi (2)^2}{4} * 1.0 * 0.2 * 10 = 6.28$ ton ↙

$R_b = 2 * \left[\begin{array}{l} t_f\ HEA(550) = 2.4 \\ t_p = 1.0 \text{ assumed} \end{array} \right] * 0.8 * 5.2 = 8.3$

$e = 2\phi$

$T = \frac{0.75 M_x}{0.8(h - t_f)} = \frac{0.75 * 63.99 * 100}{0.8(54 - 2.4)} = 116.3$ ton

→ top plate only

$n_3 = \frac{116.3}{6.28} \cong 20$ bolts

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$n_3 = 20$ b

Design of top plate

$f = \frac{T}{A_{net}} \ngtr 2.1$

$A_{net} = [b_f - n_c * \phi] * t_p = [30 - 4 * 2] t_p = 22$

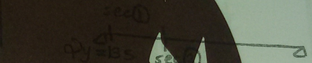
$f = \frac{116.3}{22 t_p} \ngtr 2.1$ $t_p = 2.6$ cm

ⓑ Design a welded plate girder section for an intermidate cross girder for the following straining actions

	S.A	D.L	L.L+I	total S.A
sec ①	shear	35	100	135 ton
sec ②	shear	20	60	80 ton
	Moment	80	250	330 t.m

الرسالة

Ⓐ approximate design



$Q_y = 135$

$Q_y = 80$ ton

$M_x = 330$ t.m

① web plate

$$\cdot\ d = (0.25 \rightarrow 0.3)\sqrt[3]{\frac{330}{2.1}} = 1.4 \rightarrow 1.6$$

$$\cdot\ \boxed{\text{try } d = 1.5\text{m}}$$

$$(t_w)^2 = \frac{Q_y}{41.65\sqrt{F_y}} = \frac{135}{41.65\sqrt{3.6}} = 1.7$$

$t_w = 1.3$ cm

② Flange plate

$$T = C = \frac{M}{d} = \frac{330}{1.5} = 220\,ton$$

$$A_f = \frac{T}{2.1} - \frac{1}{6} d * t_w$$

$$A_f = \frac{220}{2.1} - \frac{1}{6} * 150 * 1.3 = 72.3\,cm^2$$

$$b_f = (0.2 \longrightarrow 0.3)d = 30 \longrightarrow 45c$$

take $b_f = 40\,cm \longrightarrow t_f \cong 2.0\,cm$

Check $\frac{(b_f - t_w)/2}{t_f} = 9.675 < 21/\sqrt{3.6} = 1$

ok

الرسالة

Ⓑ Check of Buckling due to shear

① case of No V.S stiffener

$$d/t_w = 150/1.3 = 115.4 > 159/\sqrt{F_y} = 83$$

3.6

$$q_b = \left[\frac{119}{\left(\frac{d}{t_w}\right)\sqrt{F_y}}\right] * 0.35F_y = \left[\frac{119}{115.4 * \sqrt{3.6}}\right] * 0.$$

$$q_b = 0.685\,t/cm^2$$

$$q_{act} = \frac{Q_y}{A_{web}} = \frac{135}{150 * 1.3} = 0.69\ t/cm^2$$

$q_{act} > q_b$

$\therefore$ (unsafe)

② check of shear in case of use V.S stiffner

take $d_1 = 1.5m$

$$\alpha = \frac{d_1}{d} = \frac{1.5}{1.5} = 1.0$$

$$k_q = 5.34 + 4 = 9.34$$

$$\lambda_q = \frac{d}{t_w} \sqrt{\frac{F_y / \sqrt{3}}{1898\, k_q}}$$

$$\lambda_q = \frac{150}{1.3} \sqrt{\frac{3.6/\sqrt{3}}{1898 * 9.34}} = 1.25 > 1.2$$

$$q_b = \frac{0.9}{\lambda_q} * 0.35 F_y = \frac{0.9}{1.25} * 0.35 * 3.6 = 0.9072\ t/cm^2$$

$q_{act} = 0.69 < q_b$

ونقدر نصغر التخانة (tw)

الرسالة

© Check of Buckling due to Bending

$d/t_w = 150/1.3 = 115.$ $> 190/\sqrt{F_y} =$
$\underset{3.6}{F_y}$

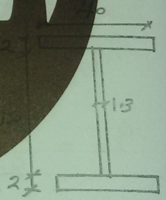
Use H.S stiffner at (0.2d) from comp flan

$d/t_w = 115.4 < 320/\sqrt{F_y} = 168.7$

∴ safe in case of use H.S stiffner at (0.2d) from comp flange

الرسالة

ⓓ check of Bending stresses



$I_x = \frac{1.3*(150)^3}{12} + 2*40*2*(76)^2$

$I_x = 1289785\ cm^4$

$y_{max} = 77\ cm$

$f_t = \frac{M_x}{I_x} y_{max} = \frac{330*100}{1289785} * 77 = 1.97\ t/cm^2$

$q_{act} = \frac{Q_{sed}}{d*t_w} = \frac{80}{150*1.3} = 0.41\ t/cm^2$

$f_{all} = \left(0.8 - 0.36\frac{q_{act}}{q_b}\right) * f_y = \left(0.8 - 0.36\frac{0.41}{0.907}\right) * 3.6$

$= 2.29 > 2.1 \rightarrow f_{all} = 2.$

$f_t = 1.97 < f_{all}$ safe

ⓔ check of lateral torsional Buckling during construction

$M_{ll+I} + M_{ol} = M_x$ في حالة كل الأحمال

يكون $L_u = zero$ نتيجة وجود (cover) الكمرة

ولا يتم عمل الـ (check) في مرحلة التنفيذ

$L_u = 1.5 m \longrightarrow M = M_{ol} = 80 t.m$

$L_{u1} = \frac{20 b_f}{\sqrt{F_y}} = \frac{20 * 40}{\sqrt{3.6}} = 421.6 cm$

$L_{u2} = \frac{1380 A_f C_b}{d * F_y} = \frac{1380 * 40 * 2}{150 * 3.6} = 204.44 cm$

$L_u < L_{u1}$

$< L_{u2} \longrightarrow$ No lateral torsional Buckling

ⓕ check of Fatigue stresses

$f_{fatigue} = \frac{M_{ll+I}}{I_x} * y_{max} = \frac{250 * 100}{1289785} * 77 = 1.49$

$< F_{sr}$

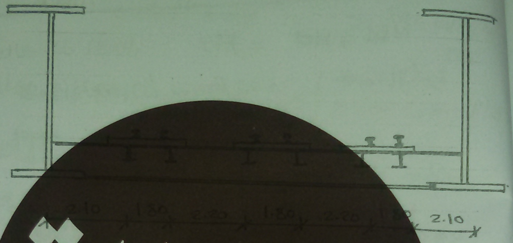
classe (B')

$\longrightarrow F_{sr} = 1.52 t/cm^2$ (ok)

No of cycles = 500 000

الرسالة

E Xame

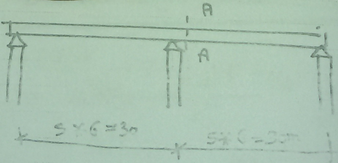

2.10
1.80
2.20
1.80
2.20
1.80
2.10

for the shown Railway through Bridge

① Design Continous stringer as rolled section.

② Design Cross girder as plate girder if B.F is used

③ Design Main Girder as plate Girder for the following S.A at sec. (A-A)


A
A
5×6=30
5×6=30

$M_{D.L} = 200$ t.m

$M_{L.L} = 500$ t.m

① DR = 30 t

② LR = 60 t

④ DL = -20

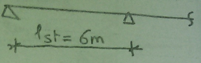
① LR = -50

الرسالة

① stringer

* statical system

Continous



بس هنا على ، نحلها simple ثم نضربها x factor

* **Loads**

D.L, L.L, I, lateral shock

* Dead load = o.w of str + o.w of track/2

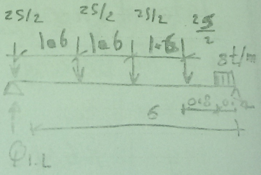
$= 0.15 + \frac{0.6}{2} = 0.45$ t/m

$M_{DL} = \frac{0.45 * l_{st}^2}{8} = 2.02$ t.m

$Q_{DL} = \frac{0.45 * l_{st}}{2} = 1.35$ t

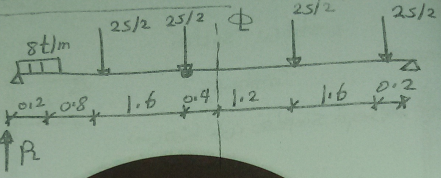
* **Live load**

Max. Q


25/2
25/2
25/2
25/2
1.6
1.6
1.6
8t/m
6
0.8
0.4
$Q_{L.L}$

$Q_{LL} = 30.11$ ton

Case of max Moment


8t/m
25/2
25/2
℄
25/2
25/2
0.2
0.8
1.6
0.4
1.2
1.6
0.2
R

الرسالة

R = 23.24

$M_{LL} = 23.24 * 2.6 - (25/2) * 1.6 - 8 * 0.2 * 2.5 = 36.4$ t.

$I = \phi - 1.0$

$\phi = 0.73 + \frac{2.16}{\sqrt{L_I} - 0.2}$

L_I continous $= 3L_{st} = 3*6 = 18$

$\phi = 1.264$

$I = 1.264 - 1.0 = 0.26$

L_I single $= L_{st} + 3 = 9.0$

$\phi = 1.5$

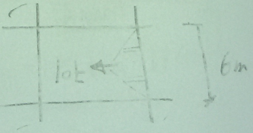
$I = 0.5$

lateral shock

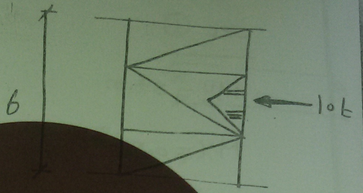
* If No Bracing

$Q_x = 10$ ton

$M_y = 10 * 6/4 = 15$ t.m



* If use lateral shock Bracing

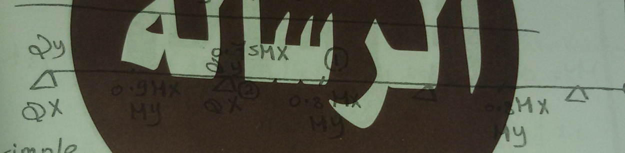


$Qx = 10$ ton

$My = \frac{10 * 2}{4}$

$My = 5$ t.m

final straining actions


Qy
0.75MX
Qy
①
0.9MX
My
②
QX
QX
0.8 MX
My
0.8MX
My

الرسالة

simple

$Mx = 2.02 + 36.42 * (1 + 0.5) = 56.65$ t.

$My = 5$ t.m

$Qy = 1.35 + 30.11 (1 + 0.5) = 46.52$ ton

$Qx = 10$ ton

continous

sec ①

$My = 5\,t.m$

$Mx = 0.8\left[2.02 + 36.42 * (1+0.26)\right] = 38.$ t.

sec ②

$Qy = 1.35 + 30.11\,(1+0.26) = 39.3\ ton$

$Qx = 10\ ton$

$Mx = 0.75\left[2.02 + 36.42 * (1+0.26)\right] = 35.93\ t.m$

ويتم تكملة التصميم مثل امتحان (2012)

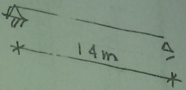
مع ملاحظة

$lu = 2.0\,m$

الرسالة

Cross Girder

statical system



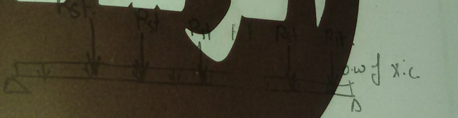
loads D.L, L.L, B.F

Dead load

الرسالة

0.6 → c.f [illegible] rolled sec.

→ 0.[illegible] thin plate Girder

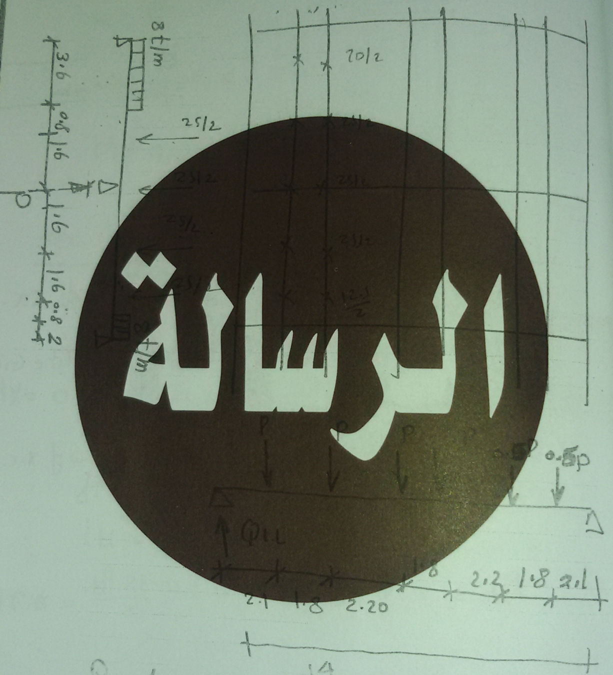

Rst.
Rst
Rst
Rst
Rst
o.w of x.c

$R_{st.} = 2\ Q_{D.L}|_{str.} = 2 * 0.45 \frac{l_{st.}}{2} = 2.7$ ton

$M_{D.L} = \frac{0.6 * 14^2}{8} + 2.7(1.1 + 3.9 + 6.1)$

$= 34.3$ t.m

$Q_{D.L} = \frac{0.6 * 14}{2} + 3 * R_{st.} = 12.3$ ton live load


8 t/m
3.6
0.8
1.6
1.6
1.6
1.6
0.8
2
25/2
20/2
25/2
25/2
25/2
25/2
25/2
25/2
12.5/2
P
P
P
0.5P
0.5p
Q11
2.1
1.8
2.20
1.8
2.2
1.8
2.1
14

الرسالة

$P = 17.67 + 12.5 + 17.8 = 47.97$ ton

$Q11 = 133.6$

$Mx =$

impact

$I = \phi - 1.0$

$\phi = 0.73 + \frac{2.16}{\sqrt{LI} - 0.2}$

$LI = 2 * 14 = 28$

$\phi = 1.15 \longrightarrow \boxed{I = 0.15}$

* B.f

if using B.f Bracing

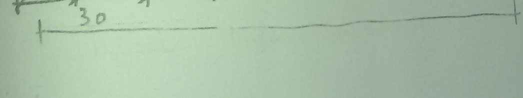
$M_y = 0$

$Q_x = 0$

ولو الامتحان لم يذكر [illegible] الاسهل نفرض انه فيه B.f Bracing

if no Bracing used

وهو ما ذكره في هذا المثال


25
25
25
25
8 t/m
1.6
1.6
1.6
0.8
30

الرسالة

From table

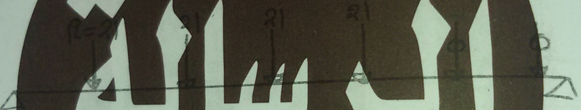
$L_o = 2L_{st} = 2 \times 6 = 12 > 7.0$

$B.F = 25 + 2(L_o - 7) = 25 + 2(12 - 7) = 3$

$D.F = 30 + 2.4(L_o - 7) = 30 + 2.4(12 - 7) =$

take larger of B.F , D.F

get $P = D.F/2 = 42/2 = 21$ ton



get Q_x = ✓

M_y = ✓

الرسالة

Design of X.G (Plate Girder)

if B.F Bracing is used

$M_y = 0$

$M_x = 513$ t.m

$Q_y = 131$ ton

Approx. dimensions — web / flange

hw $h_w = 0.3 \sqrt[3]{\frac{513}{2.1}} = 1.90$

tw $q = \frac{Q}{190 \times t_w} \leq 51.26$

get $t_w = \ll$

$$t_w = \sqrt{\frac{Q}{41.65\sqrt{3.60}}} = 1.29 \text{ cm}$$

Final Taken $t_w = 1.2$ cm

الرسالة

Flange

$$T = \frac{M}{h_w} = \frac{51300}{190} = 270 \text{ t.m}$$

$$A_f = \frac{T=270}{2.1} - \frac{h_w * t_w}{6} = 90.5 \text{ cm}$$

$$b_f = (0.2 - 0.3) \overset{190}{h_w} = 45 \text{ cm}$$

38 57

$$t_f = \frac{90.5}{45} = 2.2 \text{ cm}$$

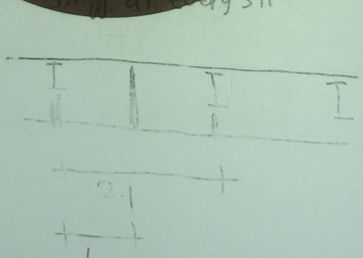
$$\frac{b_f/2}{t_f} = \frac{45/2}{2.2} = 10.20 < \frac{21}{\sqrt{f_y}}$$

o.k

الرسالة

check Buckling of web due to Q

Put V. stiff. at every str.



$d_1 = 1.10$ m.

$$\alpha = \frac{d_1}{d} = \frac{110}{196} = 0.6 < 1.0$$

$$k_q = 4 + \frac{5.34}{\alpha^2} = 18.80$$

$$\lambda = \frac{(d/t_w)}{57}\sqrt{\frac{F_y}{k_q}} \quad \left(\frac{190}{1.20}\right) \quad \frac{3.60}{18.80}$$

$$= 1.21$$

الرسالة

$$q_{rd} = \left(\frac{0.90}{\lambda_q}\right) 0.35\, f_y = 0.93\ t/cm^2$$

$$q_{act} = \frac{Q}{h_w t_w} = \frac{131}{190 * 1.20} = 0.57\ t/cm^2$$

chelk Bucklig of web due to M

$$\frac{h_w}{t_w} = \frac{190}{1.20} = 158$$

$$\frac{\overset{100}{\cancel{190}}}{\sqrt{F_y}} < \frac{h_w}{t_w} < \frac{\cancel{320}}{\sqrt{F_y}}$$

168

use Th.st

check Bending stresses

≡ check fatigue

$$f_{act.} = \frac{M}{I_x} y_{max.} \qquad \frac{h_w}{2} + t_f = \frac{190}{2} + 2.20$$

$$1.20 * \frac{190^3}{12} + 2 * 45 * 2.20 \left(\frac{190}{2} + \frac{2.2}{2}\right)^2 = 2514471 \text{ cm}^4$$

الرسالة

$$f_{act.} = \frac{513 * 100 * (95 + 2.20)}{2514471} = 2.04 \text{ t/cm}^2$$

< f_{ball}

f_{ball}
Compact
non Compact

$L_u = 2.20$ (dist. bet str.)

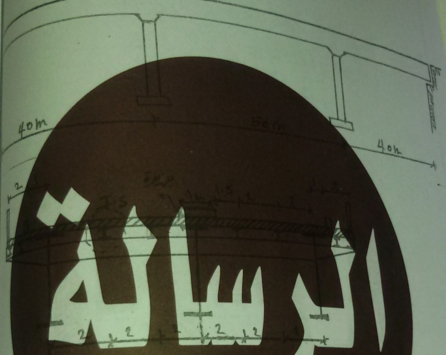
$$L_u \leq \frac{20 b_f}{\sqrt{F_y}}$$

$$\leq \frac{1380 A_f C_b}{F_y * h}$$

ok

$f_{ball} = \frac{2.1}{s.f.c}$

prob ⑥ Final (2002)



الرسالة

. Four lane Roadway continousplate girder Bridge

floor = 25cm R.C ، distance between stringers $d_1 = 2m$

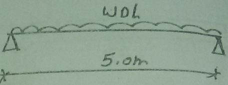
. stringer running over cross girder (continous)

distance between X.C = 5.0m

. وجود الجزيرة فى منتصف الطريق فصل الطريقين ولكن مع ذلك نوجد (main lane)
واحدة و (sec lane) واحدة وهذا لا يخلف شئ من التحميل للاحمال بالاضافة
ان أول عجلة تحميل تبعد عن الجزيرة (0.5m) ، كما تبعد عن الـ sidewalk (0.5m)

ⓐ Design a rolled sec for an intermidate stringer

① Dead load



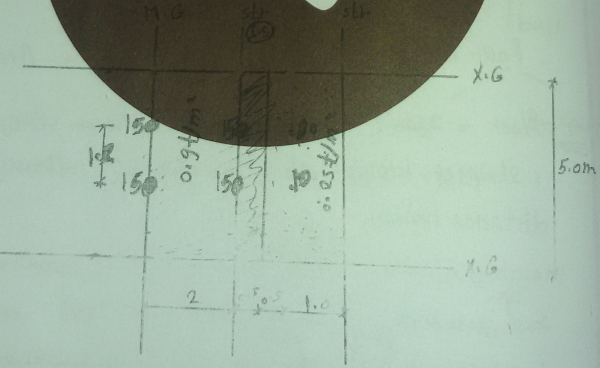
$W_{DL} = 0.1 + (2.5 * 0.25 + 2 * \frac{5}{100}) * 2$

$W_{D.L} = 1.55\ t/m$

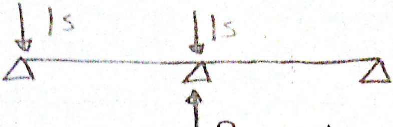
$Q_{DL} = 1.55 * 5/2 = 3.875\ ton$

$M_{D.L} = 1.55 * (5)^2/8 = 4.84\ t.m$

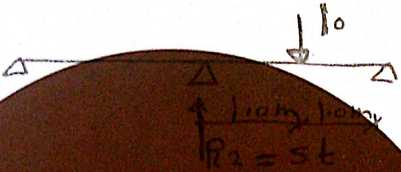
② live load


M.G
str
str
1.5
X.G
150
150
150
150
10
10
0.9 t/m²
0.25 t/m²
5.0m
X.G
2
0.5
0.5
1.0

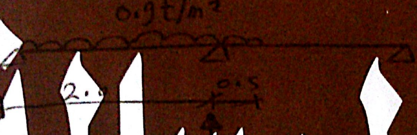
الرسالة



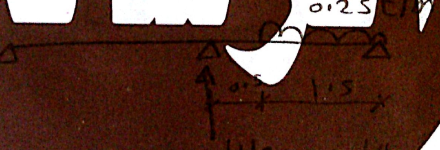
$R_1 = 15\,t$



$R_2 = 5\,t$

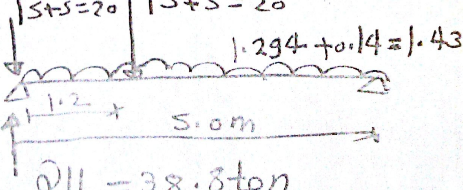


$W_1 = 0.394 + 0.9 = 1.294$


0.25t/m²
0.5
1.5

$W_2 = 0.14$

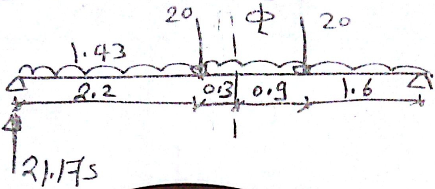
Case of max shear


15+5=20
15+5=20
1.294+0.14=1.43
1.2
5.0m

$Q_{LL} = 38.8\,ton$

الرسالة

Case of max moment


20
℄
20
1.43
2.2
0.3
0.9
1.6
21.175

$$M_{LL} = 21.175 * 2.2 - 1.43 * 2.2 * 1.1 = 43.12\ t.m$$

Final straining actions

$$M_X = 4.84 + 43.12 = 47.96\ t.m$$

$$Q_y = 3.875 + 38.8 = 42.675\ ton$$

الرسالة